Foundations of Structures — (B) Piling and Piled Foundations

Fondations — (B) Les pieux et les fondations sur pieux

General Report

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Introduction

The pile foundation problem is one of the most complex problems in soil mechanics and foundation engineering. Soil deposits in the field may present unique conditions for every pile job. Therefore, the design of piles will be always one that requires experience ans sound knowledge of the behaviour of subsoil materials before and after pile driving. Empirical or theoretical methods alone are not likely to give the proper answer in many cases. Working hypothesis in the development of theories should be in agreement with field observations on the mechanical behaviour of different types of materials under the pile tips and in the neighborhood of the piles shaft.

The pile foundation problem should be divided in two : the stability and the elasticity problem. The first problem is the solution to avoid the pile or pile group to break into the ground. The second problem is the solution concerning total and differential settlements of a pile or pile groups to avoid damage to the superstructure. The second problem is by far the most common in practice. Excessive settlement of piles and pile groups are always necessary to predict and control.

The stability problem is understood by the ultimate load capacity of a pile : a) the point bearing capacity and, b) the ultimate friction load capacity.

The point bearing capacity is known to be a function of the geometrical dimensions, shape and roughness of the point, confining pressure at the point as well as shearing-strength, density and compressibility of the soil under and around the point of the pile after driving. In case of caissons or bored piles the possible alteration of the mechanical properties of the material during excavation should be considered.

The ultimate friction load capacity is a function of the shearing strength of the soil, density and compressibility after pile driving. In clays the time element is important. The horizontal effective stresses remaining after pile driving, because of volume displacement, are important as well as the compaction or consolidation of the soil around the pile. The surface of failure may be located at different distances from the pile shaft depending on the type of material and change of mechanical properties after driving the pile. In clayey soils the remolded material because of pile driving consolidates against the pile shaft gaining higher shearing strengths. Thus, the surface of sliding in most cases is not at the interface between soil and pile. In clays the sensitivity is a very important factor as for clays of different sensitivity the shear surface may be at a different locations from the pile shaft.

High sensitive clays never regain their total lost strength due to damage during pile driving. Therefore, a minimum strength is encountered at certain distance from the pile shaft. In some cohesive soils, cohesion may be entirely broken during pile driving remaining only the friction. Therefore, it is imperative to learn more on the real horizontal effective pressures remaining after pile driving for different subsoil materials.

Empirical methods based on penetration devices have been in most cases the only means to estimate the load capacity of piles. However, these methods are highly dependent on the size of the device used in conjunction with subsoil materials. Therefore, in many instances large deviations may be expected in their practice. On the other hand they are very valuable tools in areas where experience has been gained by their application during a great number of years.

The adjustment of rational methods to estimate the load capacity and settlement behaviour of a pile and group of piles is necessary in order to be able to predict with better accuracy the behaviour of pile foundations within the most possible economy and without sacrificing safety.

It is obvious that in great many cases of difficult subsoil conditions the empirical methods by means of penetration devices are not powerful enough and careful properties of shearing strength and compressibility of the materials as well as hydraulic conditions have to be secured to achieve a rational and more successful pile foundation design.

Many papers may be found in the literature on pile foundations dealing with different subjects in the same paper, thus making a difficult task for classification in the aim to find out the most valuable points discussed. Therefore, the general reporter feels that it is necessary to study a classification of subjects in such a way that the authors in the future will confine their reports adding experience and understanding to a definite problem under investigation.

For the time being the general reporter finds convenient the following classification :

(a) Laboratory investigations on model piles and related soil properties.

(b) Investigations of natural size pile tests and related subsoil properties.

(c) Theoretical investigations on the behaviour of piles and related experiments.

(d) Investigations of the behaviour of pile foundations and related subsoil engineering characteristics.

(e) Investigations on special piles and pile foundations.

(f) Correlations between driving resistance of piles with subsoil investigations.

The discussion of every one of the points mentioned above are out of the scope of this general report, however, some comments may be stated briefly.

Laboratory investigations are always very valuable and general laws of behaviour of model piles may be discovered. Nevertheless care should be taken to investigate only that phenomena that has a true similitude with same phenomena in the field. Otherwise the efforts made in laboratory investigations not following similitude laws under same mass and dynamic forces may become only of academic value.

Theoretical investigations are of important value when the basic working hypothesis are in agreement with experimental results in the field or dynamically similar tests in the laboratory. Many of the theories developed up to date can not be properly supported because of the lack of sufficient field evidence.

Pile tests in the field may give the information on the ultimate bearing capacity of a pile. The method of testing is sometimes not carefully explained, as there are different ways of testing a pile and of the interpretation of the results. Each pile test investigation should be accompanied with all possible information about stratigraphy, hydraulic conditions of the site before and during pile testing and mechanical properties of the materials before and after pile driving.

The investigation of pile foundations in service in conjunction with a soil mechanics and geological interpretation is one of the most powerful information to learn about the behaviour of pile groups. Careful settlement records should be accompanied in conjunction with methods of construction and load history.

In accordance with above short discussion on pile foundations and with the point of view of finding an advancement towards a better understanding on the behaviour of pile-foundations, the general reporter is in the position to classify and report briefly the valuable work of 28 authors submitting papers to this Congress in the aim to increase knowledge to this important and interesting topic on the mystery of the behaviour of pile foundations.

Eight papers fall into investigations on model piles and related soil properties, twelve on pile tests and related subsoil properties, five on theoretical investigations and three on special piles and pile foundations for buildings.

Laboratory investigations on model piles and related soil properties

Messrs. G. F. Sowers, C. B. Martin, L. L. Wilson and M. Fausold, Jr., U.S.A., presented an interesting investigation on the load capacity of friction pile groups in homogeneous clay. They reached the conclusion that the point bearing capacity is small and depends on the compressibility of the clay. They used a mixture of commercial bentonite clay and water with 300 per cent water content and sensitivity of 2. The bearing capacity coefficient N_c was found about 5 instead of 9·5 as given by theory. The skin friction becomes uniform when failure.

The authors tested groups of 2, 4, 9 and 16 piles. For spacings larger than 2 diameters invariably the pile group efficiency was obtained on the order of 0·8 for all the lengths of piles tested, that is : 2, 24 and 36 diameters long.

Messrs. M. R. Saffery and A. P. K. Tate, U.S.A., have presented the results of an interesting investigation on the subject of ultimate load on group of piles when the load is applied excentrically. They have performed model tests on London remolded clay. The authors used a value $N_c = 9·5$ to deduct point bearing from the ultimate load on single piles of different lengths, thereafter they found values for the reduction coefficient of 0·51 to 0·56. In the conclusions it is interesting to note that for a friction pile, when the toe-bearing is less than 20 per cent, the efficiency of the group for any given spacing appears to be independent of pile length. Excentric loading does not affect the ultimate load capacity of the group and increases from 80 per cent for about 2 diameters to 90 per cent at 4 diameters and more.

Mr. T. K. Chaplin, United Kingdom, reports an interesting study on compressibility of sands and settlements on model footings. Particularly interesting are the results he reports in Figs. 2 to 6 where the author invariably obtains straight line relationships between the volumetric strain and the square root of the applied pressure. It would be gratifying if Mr. Chaplin would present during discussion the explanation of his nomenclature and briefly explain how the different tests were performed as presented in the graphs. The mass forces confining sand deposits in the field are extremely important. Theoretically to compute settlements it would be necessary to learn the compressibility at the state of stress in the field added by that produced by the weight of the foundation.

Messrs. J. G. Stuart and T. H. Hanna, United Kingdom, have performed an investigation on two parallel strip footings at different spacings, for surface footings and deep footings. The authors found that for surface footings efficiency factor increases to a value on the order of 2 when the strip footings approach each other to a spacing equal to the width of the footing, and a value of one as the spacing increases to about 5 times the width of the footing. However, for deep foundation the arching effect between the side-walls plays an important part such that for close spacing the system has the tendency to work as a single unit. Nevertheless, the authors find a maximum efficiency on the order of 1·3 for a spacing between 2 and 4 and depth ratios of 8, 16 and 20.

Messrs. R. Haefeli and H. Bucher, Switzerland, presented a method to estimate from model tests the permissible load and settlement of piles. A theory was developed to compute the effect of settlement of the point load and the settlement from friction load. The law of deformation may be adjusted in the proposed theory and for certain settlement of the pile the share of load taken by the point and skin friction may be computed. The authors reach to interesting conclusions.

Dr. A. K. Gamal Eldin, Egypt, has reported interesting results of pile model tests in the aim to clarify the question why the ultimate load capacity of a friction pile as computed from the classical theories of point resistance and skin friction along the shaft give values below the real ones. The author proposed a new formula that is in good agreement with the results of his investigation.

Messrs. T. Mogami and H. Kishida, Japan, report an investigation to determine the stress distribution in the soil to a certain distance under the point of the pile. They find that the theory of elasticity gives good agreement in case of sand when assuming point load applied at the tip of the pile instead at 1/3 of the tip as usually used. Concluding that most of the load goes to the tip of the pile.

Field tests showed to the authors that penetration force in the field is proportional to penetrating depth contrary to what observed in the laboratory. Furthermore, the authors found from pulling field test in piles driven in Tokyo and Yokohama clays that the ultimate load divided by the embedded surface area of the pile is nearly equal to the cohesion of the clay and the shearing stress is distributed uniformly as deduced from the measured straight line distribution of stresses along the pile.

Moreover, the authors found that the displacement stress in sand is a very important factor in piles driven in sand with spacings less than two diameters and no effect is observed over 4 diameters. Superposition law of stresses may be applied for piles with spacings over three diameters.

Dr. C. Szechy, Hungary, describes an interesting investigation performed to find out bearing capacity of model tubular piles driven in sand with and without bottom. The related compaction of the sand during driving was also measured. For a depth ratio greater than 10 the author found no appreciable difference in bearing capacity between open and solid bottom piles.

Investigation of natural size pile tests and related subsoil properties

Prof. J. KERISEL, France, has presented a comprehensive study on the bearing capacity of piles in compact and very compact sand. The author investigated the point resistance and the lateral friction along the shaft of the pile. The tests were performed on natural scale in a reinforced concrete container 6·4 mts diameter and 10·25 mts depth where fine sand of the Loire was carefully placed and uniformly compacted to a dry unit weight of 1·75 t/m^3. From the results of his experiments and the analysis of reports on the penetrometer by himself and other authors he reached very interesting conclusions given in Figs 8 to 11.

Concerning point bearing he finds that the bearing capacity factors do not depend only on the angle of internal friction. Furthermore, the author found that under the point of the pile there is a zone on the order of one diameter away from the pile shaft and enclosing the point where the soil is highly compacted. However, close to this zone the author found a channel where the sand changed its density to a smaller value than that corresponding to the original density. This channel extends only to about 2 diameters away from the pile and runs into the pile shaft to about three diameters from the point.

The general reporter concurs with the author in stating that this observation is extremely important in the development of an elastic-plastic theory in the aim to obtain better results for point bearing capacity problems in piles from mechanical properties of the soil in the field after pile driving.

Concerning skin friction this was investigated with penetrometers with shafts of different diameters. The author reached the interesting conclusion that after certain depth ratio the total force increases almost linearly with depth reaching asymptotically a unit value of skin friction on the order of 5 to 6 t/m^2. This is also an interesting result to be considered, demonstrating that the mass forces are quite important in pile tests in situ as the way the state of stresses develops around them.

Messrs. H. CAMBEFORT and R. CHADEISSON, France, have presented a method to determine the load capacity of a pile from loading tests taking into account the secondary time effect. Evidently the load capacity of a pile in clay depends on time. A short duration loading test is not representative of the real ultimate load of a pile. The law of secondary deformation is important. The authors found this law is better represented by : $\log (1 - m\sqrt{t})$ instead of the usual law taking the settlement proportional to $\sqrt{t}$ or $\log t$. The authors present an interesting example where an ultimate load capacity of 40 tons is obtained when plotting the load vs settlement curve with small increments of time as compared with the ultimate load capacity of 26 tons when deformation increments are extrapolated to 100 hrs. The result is very significative.

Messrs. R. J. WOODWARD, R. LUNDGREN and J. D. BOITANO, Jr., California, U.S.A., have presented an interesting investigation on pile loading tests in stiff clays in the aim to throw more light on the reduction coefficients to evaluate skin friction along the shaft of the pile from the unconfined compressive strength determined in undisturbed samples in the laboratory. Their final results are summarized in Figs. 4 and 5. In Fig. 5 the authors give the results obtained from tests by other investigators. From this figure it may be obtained that for a clay with shearing strength on the order of 1 K/c^2 the reduction coefficient may vary between 0·2 and 0·82. For soft to medium stiff clays the authors find a reduction coefficient reaching a value on the order of one.

F. A. SHARMAN, Sir William HALCROW and Partners, United Kingdom, have presented an investigation on the penetration resistance of friction piles in soft and very soft clays from which they computed the reduction coefficients for eleven test piles using average values of undisturbed and remolded shear strengths determined with the vane. Point bearing was deducted using $N_c = 9{\cdot}5$. For undisturbed strengths reduction coefficients were found varying from 0·2 to 1·0, and for the remolded state from 0·92 to 2·60. Finally, the authors proposed a reduction coefficient of 0·4 for straight sided concrete piles and of 1·0 for tapered timber piles. However, from the authors study it may be noticed that out from eleven piles, 6 piles give very close values for the reduction coefficient on the order of 0·6. The sensitivity of the clay is on the order of 3 to the full thickness of the deposit.

Dr. E. MENZENBACH, Germany, has presented an interesting and valuable statistical study to determine the factor of safety of point bearing piles on sand and gravel for penetration tests where the Dutch cone penetrometer was used. The author has used the results of a large number of tested piles in Holland for his investigation. The author reaches the conclusion that the information obtained from the cone penetrometer should be used in conjunction with the base area of the pile. The author arrives to a statistical formula for the permissible load as a function of the average cone resistance, the base area of the pile, and the allowable ratio of the failure to permissible pile loads.

Messrs. J. FOLQUE and Guy de CASTRO, Portugal, describe a test carried out on a group of 18 piles. The test consisted in pushing apart the piles with a horizontal force applied by means of a jack installed in the middle of the group, thus testing 9 piles at a time. Each group of nine piles has 6 battered piles with inclination of 6°. All the piles were driven to bedrock to a length of about 45 mts.

Messrs. Th. WHITAKER and R. W. COOKE, United Kingdom, propose a method of testing piles. From their experience in testing model piles in four different ways they reached the conclusion that the best method to obtain the ultimate load of a pile is that in which the pile is made to penetrate at a constant speed. The force applied at the top of the pile to maintain the rate of penetration being continuously measured. This has been referred as the CRP method and may be readily applied in the field in about 10 minutes duration for piles in clay and about in one hour for gravelly soils. The only inconvenience, the authors state, is that this test gives only the failing load, no attempt is made to determine the settlement a pile may suffer under sustained load. This is a straight forward method where it is only required to determine the load capacity under quick conditions.

Mr. N. O. ARTIKOGLU, Turkey, presents the results of loading tests on 8 piles driven in different subsoil conditions from clay to gravelly materials. The author makes the correlation between the ultimate tests loads with the results of cone penetration tests with the Dutch sounding device. His results are based on two formulae he uses to compute the point bearing capacity and the skin friction capacity.

Mr. L. BOGDANOVIC, Yugoslavia, reports the results of pushed in piles in comparison of those obtained from 36 mm cone penetrometer where skin friction and point resistance were measured. The results reported are very consistent. The author finds that a coefficient of safety of 2 is sufficient for the adopted limit of the linear load settelment ratio of 60 per cent based on the ultimate point resistance. The author concludes that if skin friction is also taken into account the factor of safety may be larger than 2.

Messrs. O. EIDE, J. N. HUTCHINSON and A. LANDVA, Norway, have presented the results of short and long term loading tests for friction tappered timber piles in clay. The authors made a very thorough investigation of the subsoil properties. A continuous profile of the shearing strength properties was obtained using cone, vane tests and unconfined compression tests. The vane results correlate rather well with unconfined compression tests results. The cone gives consistent higher values on the order of 33 per cent. The vane sensitivity obtained was on the order of 5 to 7.

Concerning loading tests they reported results Figs. 7 and 9 on load vs settlement of " short term " tests made in periods of 6 to 12 hours, with the piles resting before loading 3, 31, 71 and 799 days. The authors show clearly in Fig. 8 that the consolidation of the soil against the pile shaft plays an important part, reaching already 83 per cent of ultimate at 71 days. Extremely interesting is to observe from these tests the effect of increased rigidity, as say for 20 tons on the 71 day old pile gave a settlement of 6 mm and the 799 days old pile under the same load 4 mm. The use of the break in the *p* vs *s* curves for comparison is very significative.

Extremely interesting is to observe in the " long-term " test, Fig. 11, that as this was continued after a resting period the trend of the settlement is almost parallel to the curves obtained for the short-term tests. Unfortunately the authors did not discuss this effect, that shows a very significant behaviour related with reconsolidation and hardening effect of the clay close to the pile shaft.

Messrs. D. MOHAN and G. SINGH JAIN, India, describe results of a substantial number of tests made in piles cast in situ in a high compressible and expansive clay. The skin friction along the shaft of the pile was measured by pulling tests and substantially gave same value as measured in loading tests with no point bearing. The authors found skin friction of about 33 per cent of shear value determined from vane tests and unconfined compression tests. Deducting the measured skin friction the authors found values for bearing capacity stress vs. shearing strength on the order of 8·2, 5·7, 7·8 and 7·2 for the piles reported. However, the authors reach the conclusion that a value of 8 is more likely for the point bearing capacity factor.

It is the opinion of the general reporter that it would be very valuable if the authors could present the soil profiles, shearing strength and sensitivities for conclusions on piles reported in Table VI-a.

Mr. A. F. VAN WEELE, Holland, describes the use of the Dutch penetrometer to obtain the required penetration of driven piles. The author explain some of the inconsistencies in driving resistance and penetrometer information, as experience plays a very important part in the interpretation of the penetrometer results. The author reports actual driving graphs vs penetrometer where it may be seen clearly that the configuration of the curves correspond well in the sand layers where point resistance-plays the important part.

Very interesting in this paper is the experience of the author concerning cracking of piles during driving. For penetrations greater than 10 cm per blow cracking is produced in the concrete piles because of tension failure produced by the impact waves. The author makes recommendations to reduce this undesirable danger during pile driving.

Theoretical investigations on the behaviour of piles and related experiments

Messrs. V. G. BEREZANTZEV, V. S. KHRISTOFOROV, and V. N. GOLUBKOV, U.S.S.R., have presented an interesting study of point bearing capacity and the settlement of pile groups. Concerning the point bearing capacity they found from studies in the laboratory on dense sand that the confining pressure at the tip elevation of the pile depends on the shear developed in the mass of soil along a cylinder with radius equal to the vertical extension of the potential surface of sliding. The shear planes above the point shown in pictures taken of model piles in sand justify the working hypothesis used by the authors.

Concerning the settlement of pile groups the authors found from tests in the field for groups with spacing from 6 to 3 diameters that the settlement of the group depends upon the load transmitting area and not on the number of piles in the group. This means that the arching effect of the material makes the group to work as a single unit or pierlike unit. Conclusions are that the settlement varies as the square root of the transmitting area. The transmitting area at the tip of the pile is taken within an angle of 7° measured from the mantle of the exterior piles.

Mr. R. CHADEISSON, France, proposes a novel theoretical method to compute the settlement of a friction pile by establishing the differential equation of shear in the neighborhood of the pile mantle and assuming a law of variation of the shear force with the pile distance. The author finds after integration, an equation for the settlement as a function of shear modulus and of certain parameters to be determined by experiment. The shape of the settlement curves he obtains from tests in bored piles confronts remarkably well with the computed values, demonstrating that the shear distribution law he proposes is accurate. After knowing this law, the vertical state of stress close to the pile may be also computed. Therefore, the author is able to compute the influence of the confining vertical pressure of one pile against another. The results show that this action is not so important as usually believed.

The paper of Mr. Chadeisson, France, is a substantial contribution toward the understanding of the settlement behaviour of piles in the aim of finding a method of rational design. The procedure proposed by the author should be tested in other soils and the proper parameters found as in clay, silt and sandy soil deposits.

Messrs. H. MATLOCK and L. C. REESE, U.S.A., have presented a theoretical study concerning the behaviour of piles subjected to lateral loads and moments. Under the assumption of a constant soil modulus and rigidity of the pile the authors have compared solutions for the integration of the fourth degree differential equation by numerical procedures with the solutions obtained by means of the IBM 650 computer. Their results show fairly good agreement. The use of computers for this type of problems may save much time as compared with numerical procedures usually used to solve these problems.

Dr. Yoshichika NISHIDA, Japan, presented an interesting mathematical investigation evaluating the state of stress. Adjacent to the pile shaft the material is in the plastic state, thus the ratio of the principal stresses assume the flow number; at a certain distance from the pile the soil is in the elastic state. The vertical stress remains equal to the overburden pressure. The author assumes that the soil is pushed only horizontally and computes the radius of the compacted soil zone around a pile considering the volume change taking place during and after pile driving.

The investigation conducts the author to interesting conclusions on the group efficiency and the action of pile groups in sand and clay. The authors conclusions are in general agreement with observations in practice.

Prof. G. G. MEYERHOF, Canada, has reported results of an investigation on the point bearing capacity for pointed piles for wedge shaped points and cones. The theoretical estimates made by the author agree reasonably well with experiments in model shapes in the laboratory. The investigation was carried out with semi-smooth and rough surfaces.

The author gives graphs values of the bearing capacity factors N_c, N_q and N_i for angles from 0° to 90°. It is interesting to note that these factors remain practically constant for angles greater than 30° for rough surfaces, and increase rapidly towards zero angle. Hence it appears that after an angle of 30° friction along the cone starts to play a very important part, as for the smooth surface cones happens the contrary. This fact is very significant and the author reaches the interesting conclusion that " point resistance of piles with smooth tips decreases and with rough tips increases as the cone angle decreases ".

Another interesting item reported by the author is Fig. 6 for point resistance taken in consideration the relative density of sand. It may be seen that for angles larger than 30° and rough base, dense sand ($\varnothing = 45°$) gives about 10 times more point resistance than loose sand ($\varnothing = 35°$) and about 3 times more resistance as compact sand ($\varnothing = 41°$).

Special piles and pile foundations

Mr. J. J. Correa, México, has described a proceedure used in México City to design building foundations by using the skin friction ultimate capacity on point bearing piles. Point bearing piles are driven to a hard stratum. The building foundation is allowed to seat on the soil. The pile heads pierce through the foundation slab. Therefore, the building is allowed to settle with respect to the pile heads. Because of surface subsidence and weight of the building there is a continuous movement between building and piles, thus the piles work continuously under the ultimate skin friction load. The pile points have to support the total skin friction produced by the combined effect of the surface subsidence and the building load.

Since undoubtedly the ultimate friction load is not exactly the same for all the piles in question, the differences are taken by mechanical devices applied at the heads of the piles (Fig. 1), and to the foundation structure. Careful observations are kept to learn which devices should be operated in order to maintain the foundation structure working as designed and leveled.

The general reporter calls attention to the fact that on this dynamical foundation design care should be taken to maintain the piles under ultimate friction thus producing a continuous relative movement between building and pile heads, otherwise the building will emerge from the ground surface as observed in any other point bearing pile foundation in practice in México City.

Messrs. L. G. Soderman and V. Milligan, Canada, report an interesting procedure using electro-osmosis to increase rapidly the load capacity of piles driven into a varved clay deposit. The authors found that the load capacity after treatment was roughly doubled. The treatment affected only to about one diameter of the pile, same zone of the remolded material because of pile driving. Piles settled on the order of 1·2 inches during treatment and surrounding soil on the order of 3 inches.

Messrs. A. Luga, L. Vorobcov, I. Ten and J. Trofimenkov, U.S.S.R., describe the experimental results of tests of large capacity piles with and without enlarged bases. The authors describe the method of construction of " blow piles " and also the procedure of enlarging bases by vibro-damping concrete. The results of bearing capacity of screw anchor piles are also discussed. The authors give an empirical formula for safe load on piles, and thin-walled open caissons based on several coefficients determined from field tests. In Fig. 1 values of friction and point bearing capacity stresses may be found for different subsoil materials.

Conclusions and comments

Model Piles—For model friction piles in clay there is disagreement of how to apply the shearing strength of the clay to find out the ultimate load of the pile. Some authors affect the strength by a reduction coefficient of 0·5 and others by 1. Systematically all authors have deducted point bearing using $N_c = 9$ or 9·5, except Mr. Sowers and collaborators that are in disagreement because they measured in their tests a value of $N_c = 5$ for a remolded clay mixture with sensitivity of 2. Concerning group efficiency of model piles in clay there is fairly good agreement. Approximate straight line distribution of axial stresses in the shaft of friction piles at the ultimate load is in general agreement.

Natural Size Piles—The General Reporter has noticed that there are discrepancies on how to determine the ultimate load of a pile from loading tests. Load vs. settlement curves may assume different shapes. When a pile is tested with no point bearing i.e., in a pulling test there is no doubt as what is the ultimate load. However, when a pile is tested permitting point bearing and mostly in sandy deposits it is difficult to settle on the value of the ultimate load because the load continues to increase with penetration. However, most of the curves, even those of friction piles in clay show a distinctive curvature at certain load for which plastic deformation starts to become important. It is extremely necessary to reach to a general accepted procedure concerning the critical load or load at the break of the curve and the so called ultimate load in piles. Piles may be not equally good if ultimate loads are the same, but they may be equally good if settlements are equivalent for certain critical load.

Concerning reports on reduction coefficients in clay determined by pulling tests in piles there is a large variation reported from 0·2 to 1·0. All the authors appear to have computed these coefficients from sliding surface at interface of pile and soil. Mr. Eide and collaborators report to have noticed an increase in the quick shearing strength close to the pile shaft on the order of 2 times of that of the same shearing strength of the clay in situ. However, from computations based on in situ quick determined shearing strength the reduction coefficient appears to be on the order of 1·8 for average sensitivity of 6. From quick shearing strength in situ and sensitivity of 3, Mr. Mohan found values of 0·33 after three months rest, and Prof. Mogami found a reduction coefficient of one from pulling tests in Tokyo and Yokohama clays.

It appears to exist general agreement that disturbance of the clay because of pile driving takes place as far as one diameter away from the pile shaft and that the volume displaced by the pile occupies an annular ring close to the pile that may have a thickness on the order of 0·2 diameters depending on the compressibility of the material.

These observations may be of interest to study further the most probable surface of sliding in different types of clays and considering the most probable shearing strength properties of the clay encountered after driving, and after the pile has been resting in the clay time enough to allow consolidation and hardening. However, the surface of sliding may be not located at the same distance of the pile along its full length.

Concerning point bearing of piles in sandy deposits it appears to exist general agreement that the bearing capacity factors now in use should be carefully studied and investigated in natural size piles as they do not depend only on the angle of internal friction of the materials. The usually assumed surface of sliding may be strongly modified in position because of the shape and roughness of the point and the relative density and/or compressibility of the material around the pile point after driving. The reports of Profs. Kerisel and Meyerhof show distinctly this phenomenon.

Most of the reports on piles contain the use of the cone penetrometer to determine mainly the point resistance of piles with satisfactory results. The comprehensive statistical study of Dr. Menzenbach gives last findings that should be taken in consideration when using this tool and factors of safety to be expected. It appears that in Europe the cone penetration device has become even more popular for every days work.

Theoretical and Special Pile Foundation Reports—The papers presented on theoretical investigations are considered to be a contribution towards a better understanding of the behaviour of piles and pile groups; so are the special piles and pile foundations reported representing new views in methods to obtain better piles and pile foundations to solve problems of unusual difficulty.

Finally the general reporter wishes to congratulate all the authors for selecting the most interesting topics toward a better understanding of the behaviour of piles and pile foundations.

Recommendations for discussion

Actually the computation of the load capacity of a pile is not more precise than the accuracy that may be obtained in learning the stratigraphy of the subsoil and mechanical properties of the materials applying to certain pile or pile foundation. Therefore, the first question to consider is that related with the shearing strength and compressibility of the soil before and after pile driving.

Is the quick shearing strength in situ determined by vane, cone tests and unconfined compression tests the correct shearing strength to be used to compute a friction pile? Which procedure represents better the shearing strength of the clay to be applied to a friction pile? To what extent is the friction term important when testing a friction pile in clays and silty clays? What is the significance in normally and pre-consolidated type clay deposits?

During pile driving in clayey materials the natural shearing strength is damaged to certain extend depending on sensitivity. The way this takes place away from the pile shaft should be investigated. Consolidation and gain of shearing strength takes place in an annular ring around the pile shaft. Does this gain in strength compensate for the loss and to what distance of the pile shaft?

Relative density of cohesionless sand deposits may be determined with the use of the cone or standard penetration test devices. After pile driving this density is affected close and to certain distances away from the pile. Similar studies as those presented by Prof. Kerisel are necessary to investigate different initial subsoil conditions after pile-driving.

Pile loading tests investigations should be discussed and more experience reported. However, pile tests should be classified. The General Reporter suggests the following classification :

(a) Quickly loaded to failure.
(b) Continuously and slowly loaded to failure.
(c) Rest period and quickly loaded to failure.
(d) Rest period and slowly loaded to failure.
(e) Special loading tests.

Discussions are suggested to bring out the significance of a critical load in conjunction with settlement and that of the ultimate load. The aim is to find out a commonly accepted method or methods for different testing and the interpretation of load-settlement curves, in order that future reports on pile tests would be more consistent. In case of friction piles in clays the age of the pile before testing is important as demonstrated by Mr. Eide and collaborators in their valuable report. The excess pore pressures during testing close to the pile shaft should be measured.

The general reporter strongly believes that it is necessary to obtain more evidence to verify if the pulling test in one pile with soil under negative friction, may be equivalent to loading the pile under positive friction, as in sandy soils the difference may be very large. Negative friction may be recognized by the fact that vertical effective stresses in the soil mass have the tendency to reduce and positive friction increases the vertical effective stresses. The importance of this action should be carefully investigated and reported by means of field tests in clay and sandy soils. Theoretically the ultimate load under negative friction is different as that due to positive friction.

Furthermore, during pile driving to certain point resistance heave of the soil around a pile or pile groups takes place because of change in effective stresses and pore pressures. After driving the excess pore pressures dissipate and the heaved soil has the tendency to resume its original position, therefore excerting on the pile from the very beginning negative friction. This phenomenon should also be given careful consideration, since in many instances piles supposed to be point and friction bearing piles may be only point bearing.

Since the load capacity of piles highly depends on the engineering characteristics of the subsoil materials and on the state of effective stresses after pile driving ; the general reporter suggestion is that discussions should be oriented in the aim to throw more light towards the correction of working hypothesis in theories, based on real evidence in the field.

The only means to design successfully pile foundations in difficult subsoil conditions is the understanding of the way to apply to its full power all soil mechanics and geological factors encountered at the site under consideration.

Conclusions et commentaires

Modèles réduits de pieux. — En ce qui concerne les modèles réduits de pieux flottants dans l'argile, il y a un désaccord sur la façon d'utiliser la résistance au cisaillement de l'argile pour déterminer la charge limite de rupture du pieu. Certains auteurs affectent la résistance d'un coefficient minorateur de 0,5 et d'autres ne le font pas. D'une façon systématique tous les auteurs ont calculé la force en pointe en utilisant la formule $N_c = 9$ ou 9,5 à l'exception de M. Sowers et de ses collaborateurs qui ne sont pas du même avis parce qu'ils ont trouvé dans leurs essais une valeur de $N_c = 5$ pour un mélange d'argile remaniée, de 2 de sensibilité. Il y a un bon accord concernant l'effet de groupe pour des modèles réduits de pieux dans l'argile. Une distribution approximative linéaire des contraintes axiales dans le fût des pieux flottants à la charge limite est généralement admise.

Pieux de taille normale. — Le Rapporteur Général a remarqué certaines différences dans la façon de calculer la charge limite de rupture d'un pieu à partir d'essais de charge. Les courbes du tassement en fonction de la charge peuvent avoir des formes différentes. Quand on procède à un essai sur pieu sans " force en pointe ", c'est-à-dire à un essai de traction, la charge limite de rupture ne fait aucun doute. Cependant si l'on effectue l'essai sur pieu avec une force en pointe et particulièrement pour les sols sableux il est difficile de déterminer la valeur de la charge limite de rupture car la charge continue d'augmenter avec l'enfoncement. Cependant la plupart des courbes, même celles des pieux flottants dans l'argile, présentent un changement de courbure pour une certaine charge à partir de laquelle la déformation plastique commence à devenir importante. Il est tout à fait nécessaire de parvenir à une méthode générale commune pour déterminer la charge critique ou charge au changement de courbure de la courbe de rupture, et de ce que l'on appelle charge limite de rupture des pieux. Des pieux peuvent ne pas être aussi bons s'ils ont la même charge limite de rupture, mais ils peuvent l'être si les tassements sont équivalents pour une certaine charge critique.

Les résultats concernant les coefficients de réduction pour l'argile, obtenus à partir d'essais de traction sur pieux, varient entre 0,2 et 1,0. Tous les auteurs semblent avoir calculé ces coefficients à partir d'une surface de glissement située à l'interface du pieu et du sol. M. Eide ainsi que ses collaborateurs ont remarqué une augmentation de la résistance au cisaillement rapide, près du fût du pieu, deux fois plus grande que celle de la même résistance au cisaillement de l'argile en place. Cependant à partir de calculs basés sur la résistance au cisaillement rapide in situ, il apparaît un coefficient de réduction de l'ordre de 1,8 pour une sensibilité moyenne de 6. Avec une résistance au cisaillement rapide in situ et une sensibilité de 3, M. Mohan a trouvé des valeurs de 0,33 après 3 mois de repos, tandis que le Prof. Mogami a trouvé un coefficient de réduction de 1 à partir d'essais de traction effectués sur des argiles de Tokyo et Yokohama.

Il semble que tous les auteurs soient d'accord pour admettre que le remaniement de l'argile dû au battage des pieux se produit jusqu'à un diamètre du fût du pieu, et que le volume déplacé par le pieu occupe une couronne cylindrique contiguë au pieu dont l'épaisseur dépend du degré de compressibilité de la matière et est de l'ordre de 0,2 diamètre.

Ces observations peuvent servir à une étude plus approfondie de la surface de glissement la plus probable pour les différents types d'argile, en tenant compte de la résistance au cisaillement la plus probable de l'argile après battage et après que le pieu soit resté au repos pendant le temps nécessaire à la consolidation et au durcissement de l'argile. Cependant la surface de glissement peut ne pas se trouver à la même distance du pieu, sur toute la longueur de celui-ci.

Les auteurs sont d'accord, en ce qui concerne la force limite en pointe dans les sols sableux, pour reconnaître que les coefficients de portance employés actuellement devraient être soigneusement étudiés et examinés à l'aide de pieux de taille normale étant donné que ces coefficients ne dépendent pas seulement de l'angle de frottement interne des matériaux employés. La surface de glissement généralement utilisée, peut avoir sa position fortement modifiée par la forme et la rugosité de la pointe, par la densité relative ou par la compressibilité de la matière autour de la pointe du pieu après l'enfoncement. Les Communications des Prof. Kerisel et Meyerhof illustrent clairement ce phénomène.

La plupart des auteurs des Communications concernant les pieux utilisent le pénétromètre à cône, principalement pour déterminer la résistance à la pointe des pieux de façon satisfaisante. Le Dr. Menzenbach dans une large étude statistique donne les derniers résultats qui peuvent être pris en considération si on utilise cet appareil ainsi que le coefficient de sécurité qu'on peut en attendre. Il semble qu'en Europe le pénétromètre à cône soit de plus en plus utilisé pour le travail de chaque jour.

Théorie et Fondations spéciales sur pieux. — Les Communications relatives aux recherches théoriques tendent à apporter une meilleure interprétation du comportement des pieux et groupe de pieux. Il en est de même des Communications relatives aux fondations spéciales sur pieux où l'on présente des méthodes originales pour obtenir de meilleurs pieux et de meilleures fondations sur pieux et pour résoudre des problèmes difficiles et inhabituels.

Le Rapporteur Général est heureux de féliciter tous les auteurs pour avoir choisi les sujets les plus intéressants permettant ainsi une meilleure compréhension du comportement des pieux et des fondations sur pieux.

Sujets de discussion proposés

En fait, le calcul de la force portante d'un pieu n'est pas plus précis que ce qu'on peut obtenir en étudiant la composition statigraphique du sous-sol et les propriétés mécaniques des matériaux rencontrés par certains pieux ou certaines fondations sur pieux. Le point à discuter en premier serait donc la résistance au cisaillement et la compressibilité du sol avant et après le battage du pieu.

Il faudrait savoir si la résistance au cisaillement rapide calculée à l'aide de l'appareil à palettes, ou à partir d'essais de pénétration ou à partir des essais de compression simple est bien la résistance au cisaillement dont on peut se servir pour calculer un pieu flottant. Quelle est la méthode représentant le mieux la résistance au cisaillement de l'argile qui peut être utilisée pour un pieu flottant ? Quelle est l'importance du terme de frottement dans les essais sur pieux flottants dans de l'argile, et dans de l'argile silteuse ? Quelle en serait la valeur dans des dépôts d'argile normale ou préconsolidée ?

Pendant le battage des pieux dans des matériaux argileux, la résistance naturelle au cisaillement est en partir détruite à cause de la sensibilité. On devrait étudier la façon dont ceci se produit à distance du fût du pieu. La consolidation et l'augmentation de la résistance au cisaillement se localisent dans une couronne cylindrique autour du fût du pieu. Cette augmentation de résistance compense-t-elle les pertes, et ceci jusqu'à quelle distance du fût du pieu ?

La densité relative des sables non cohérents peut être déterminée par l'emploi d'un cône ou d'appareils de pénétration standard. Cette densité est affectée par le battage des pieux, à une certaine distance et près de ceux-ci. Des études similaires à celles présentées par le Prof. Kerisel sont nécessaires pour étudier des conditions initiales différentes du sous-sol après le battage des pieux.

Les essais de chargement de pieu devraient être discutés et il serait nécessaire d'avoir davantage de résultats expérimentaux. On devrait cependant classer les essais de pieux. Le Rapporteur Général suggère la classification suivante :

(a) Charge rapide jusqu'à rupture.
(b) Charge lente et progressive jusqu'à rupture.
(c) Période de repos et charge rapide jusqu'à rupture.
(d) Période de repos et charge lente jusqu'à rupture.
(e) Essais de chargement spéciaux.

Les discussions devraient mettre en relief l'importance de la charge critique par rapport au tassement et à la charge limite de rupture : le but serait de trouver une méthode commune acceptable ou des méthodes pouvant servir à différents essais et à l'interprétation des courbes charge-tassement de manière à rendre comparables les rapports des futurs essais de pieux. Dans le cas des pieux flottants dans l'argile l'âge du pieu avant l'essai est important ; ce fait a été démontré par M. Eide et ses collaborateurs dans leur intéressante Communication. L'excès de pression interstitielle près du fût du pieu devrait être mesuré pendant l'essai.

Le Rapporteur Général est convaincu que l'on devrait accumuler plus de preuves tendant à vérifier si l'essai de traction d'un pieu avec un sol sous frottement négatif donne un résultat comparable à ce que l'on obtient en chargeant un pieu sous frottement positif, cette différence pouvant être très élevée pour les sols sableux. Le frottement négatif se reconnaît au fait que les contraintes effectives verticales dans la masse du sol ont tendance à diminuer, tandis que le frottement positif les augmente. Ce phénomène est important et devrait être soigneusement étudié au moyen d'essais de chantiers dans des sols argileux ou sableux. En théorie, la charge limite obtenue par frottement négatif est différente de celle obtenue par frottement positif.

De plus, durant le battage d'un pieu à partir d'une certaine résistance de pointe, un soulèvement du sol autour du pieu ou du groupe de pieux, se produit à cause de la variation des contraintes effectives et de la pression interstitielle. Après battage, l'excès de pression interstitielle disparaît et le sol soulevé a tendance à reprendre sa place initiale, ceci en exerçant dès le début, un frottement négatif sur le pieu. On doit toujours penser à ce phénomène car il arrive souvent que des pieux que l'on suppose être des pieux portants par la pointe et par le frottement latéral, ne portent que par la pointe.

La force portante des pieux dépend donc beaucoup des caractéristiques mécaniques des matériaux du sous-sol ainsi que de l'état des contraintes effectives après battage. Le Rapporteur Général suggère d'orienter les discussions de façon à mettre en relief les corrections à apporter aux hypothèses de travail théoriques d'après les faits observés sur chantiers.

Le seul moyen de réaliser avec succès des fondations sur pieux dans des sols difficiles est de comprendre comment on peut utiliser au mieux tous les facteurs mécaniques et géologiques du sol du site étudié.